WARREN

The Investigation of a Reinforced
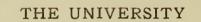
Concrete Deck Girder Highway Bridge

Civil Engineering

B. S.

1914

THE INVESTIGATION OF A REINFORCED
CONCRETE DECK GIRDER
HIGHWAY BRIDGE

BY

FRANK BAKER WARREN

———

THESIS

FOR THE

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

———

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

1914

UNIVERSITY OF ILLINOIS

COLLEGE OF ENGINEERING

May 25, 1914.

I hereby recommend that the thesis prepared under my
direction by FRANK BAKER WARREN entitled THE INVESTIGATION OF A
REINFORCED CONCRETE DECK GIRDER HIGHWAY BRIDGE be accepted as ful-
filling this part of the requirements for the degree of Bachelor
of Science in Civil Engineering.

N. B. Garver

Instructor in Structural Eng'g.

Recommendation concurred in:

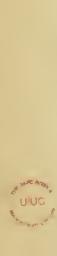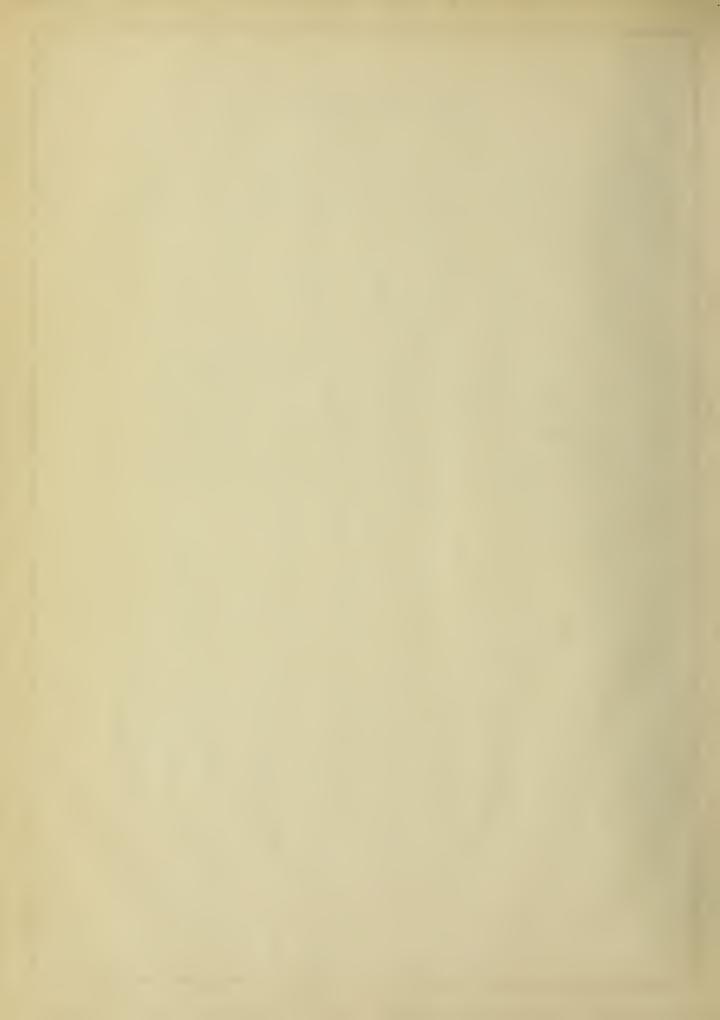*Ira O. Baker.*

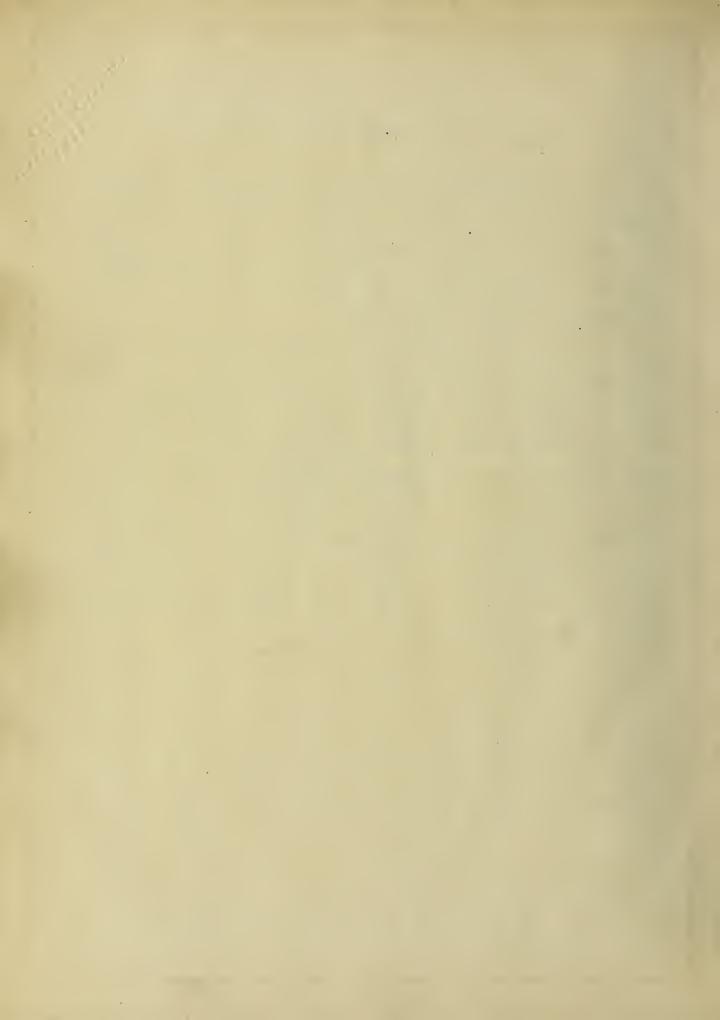Head of the Department of
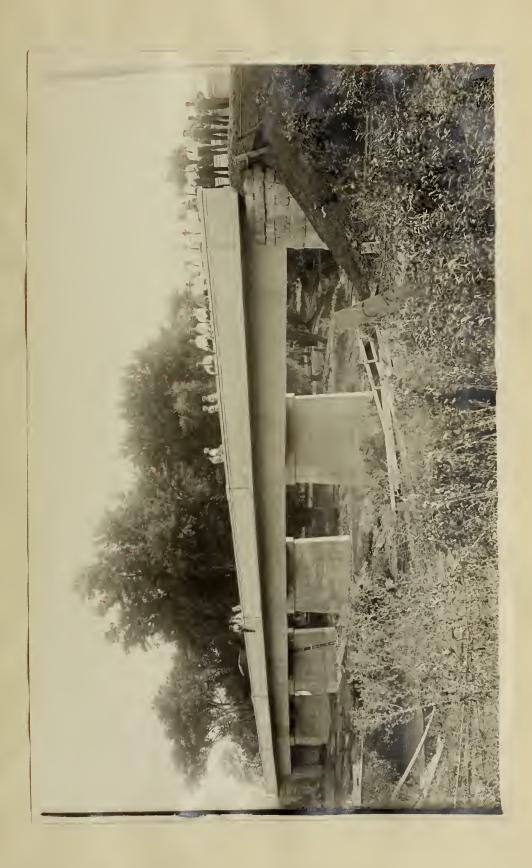Civil Engineering.

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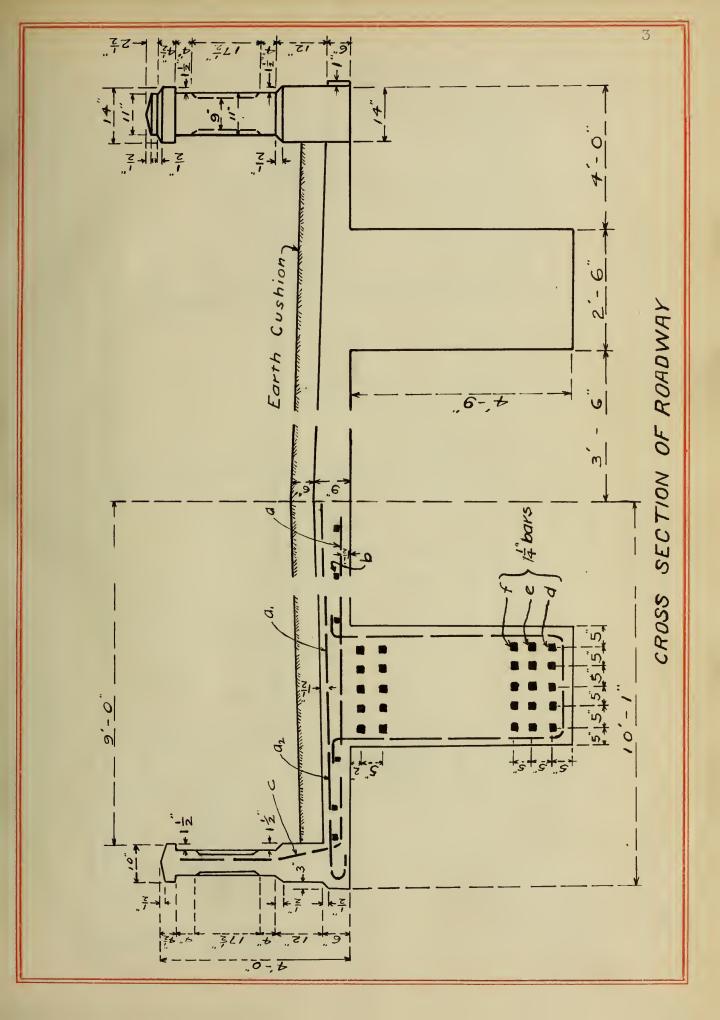
I.   INTRODUCTION.

The investigation of a reinforced concrete highway bridge was chosen as a thesis in order that the writer might become more familiar with the design of such structures, and in order that he might study the various possible conditions of loading and the stresses produced in the bridge.

The bridge chosen is of the deck girder type, having five spans of 48'- 8" clear distance between piers, (52' - 0" center to center) and a cantilever span 8' - 3 1/2" long at each end. It is located near Fisher, Illinois, over the Sangamon River, between Condit and Newcomb Townships. It was designed by F. O. Dufour, Engineer for the County Board of Supervisors, and built in 1913 under the direction of Mr. N. B. Garver, also acting as Engineer for the Board of Supervisors. Detailed drawings of the bridge and a picture of the completed structure are shown in the following pages.
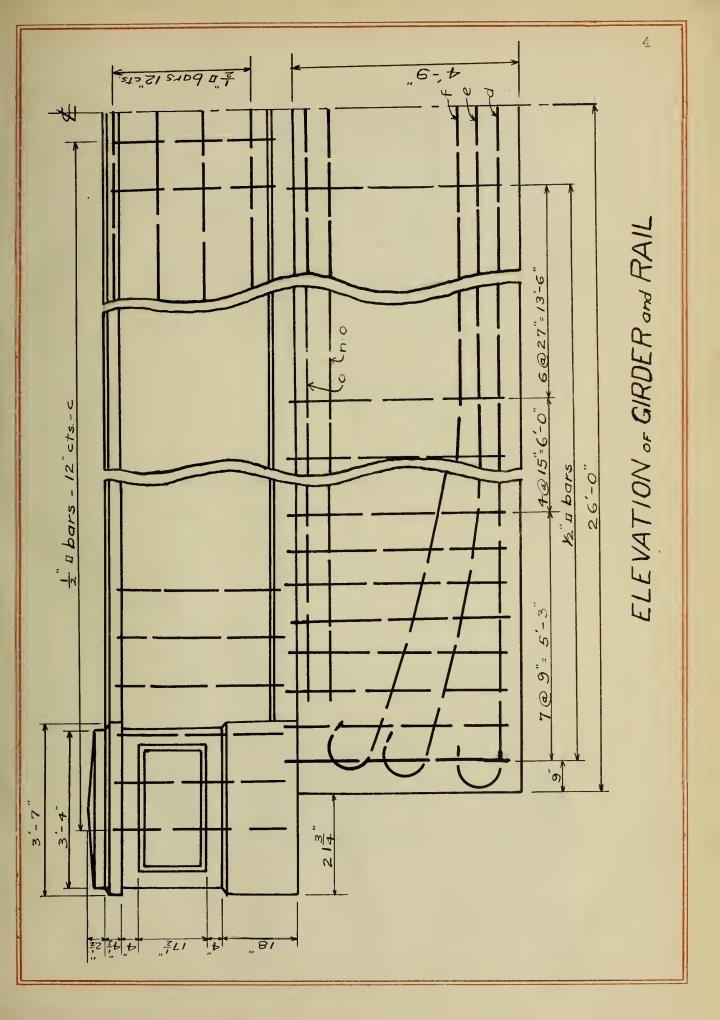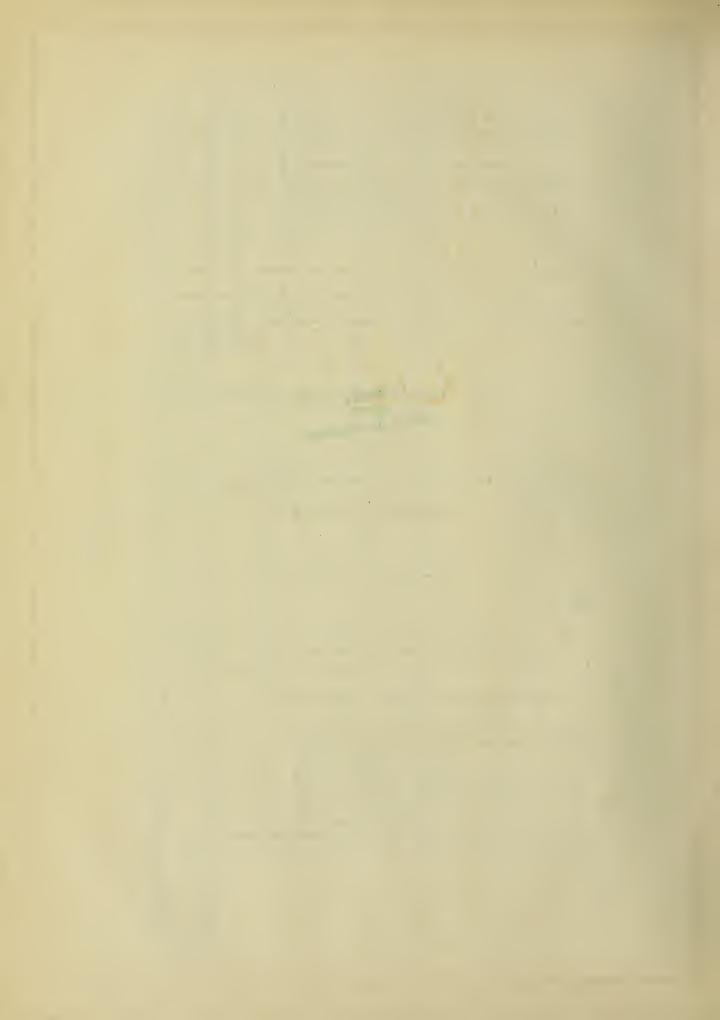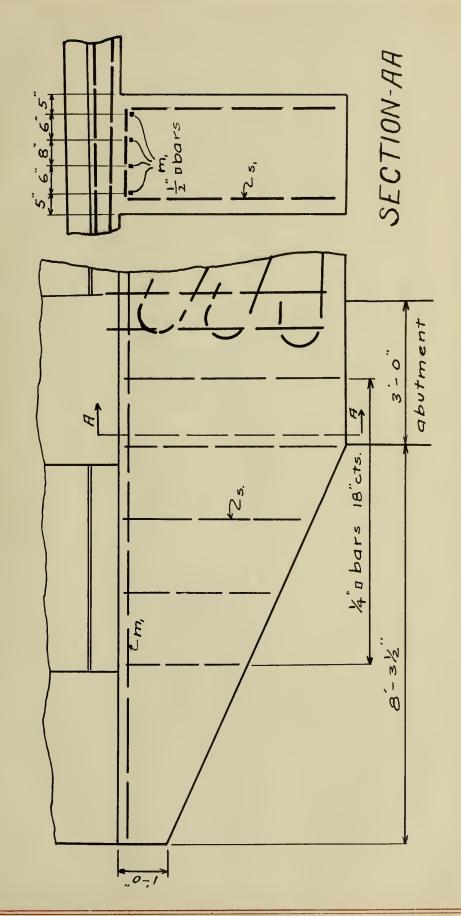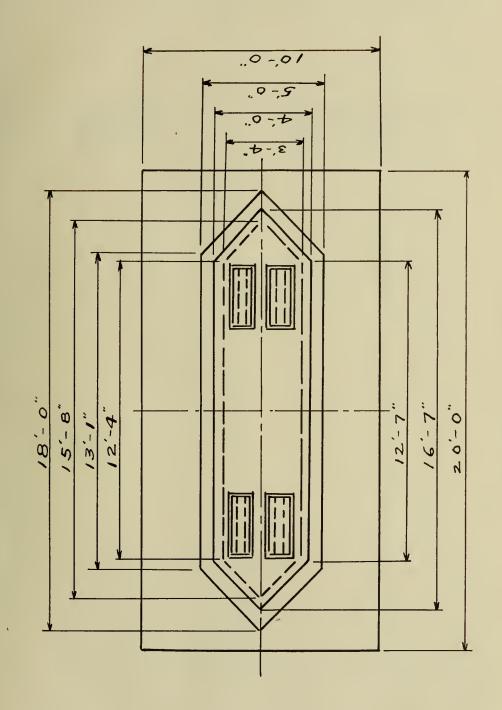
CROSS SECTION OF ROADWAY

ELEVATION of GIRDER and RAIL

HALF PLAN

SECTION-AA

ELEVATION of CANTILEVER at ENDS of BRIDGE

10'-0"

5'-0"

4'-0"

3'-4"

18'-0"

15'-8"

13'-1"

12'-4"

12'-7"

16'-7"

20'-0"

PLAN of PIER

ELEVATION of PIER
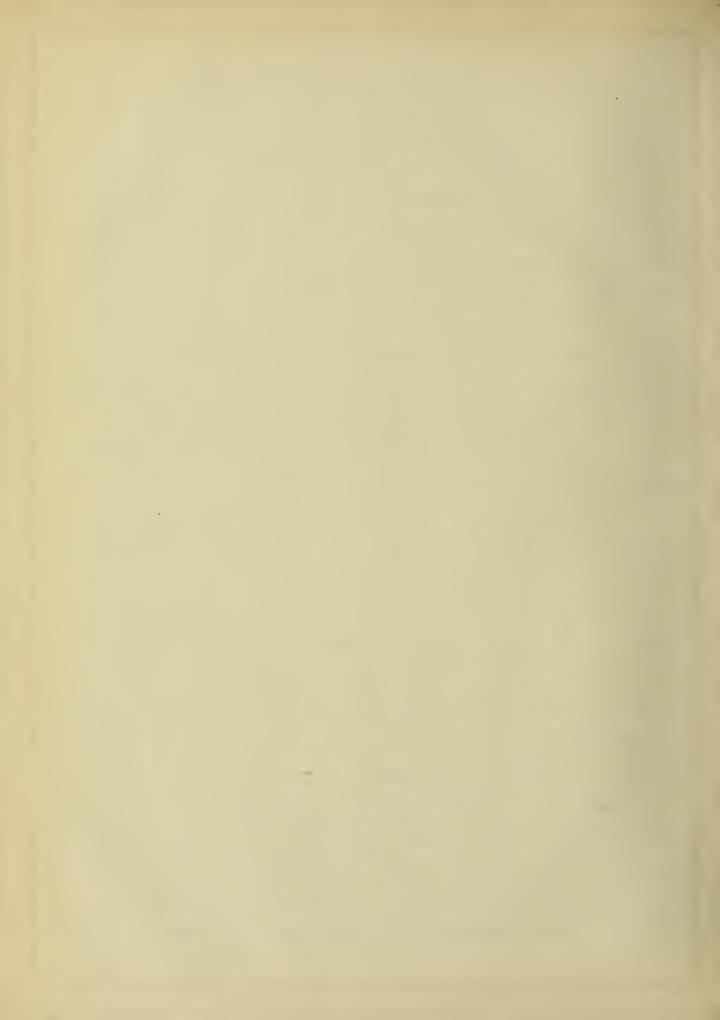
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II. BASIS OF THE INVESTIGATION.

In this investigation, the theory to be followed will be that found in "Principles of Reinforced Concrete Construction", by Turneaure and Maurer. The notation used will be taken from the above book. The value of the coefficient "n" will be taken as 15.

The dead load will be taken as the weight of the structure plus the weight of the six-inch earth cushion shown on the plans, taking the weight of reinforced concrete as 155 pounds per cubic foot and that of earth at 100 pounds per cubic foot.

The uniform live load will be assumed as 125 pounds per square foot of floor area.

The concentrated live load will be assumed as 24 tons, carried on two axles spaced twelve feet apart, the rear axle to carry 16 tons and the forward axle to carry 8 tons.
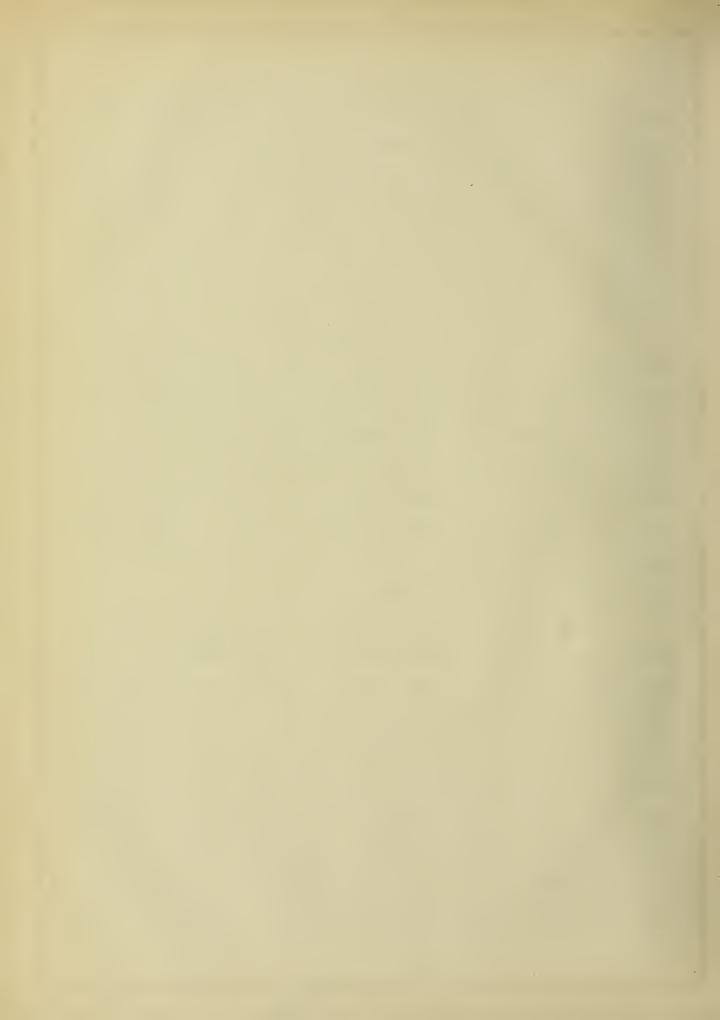
The uniform load is that used by the Illinois Highway Commission for steel spans of 50 feet. Standard specifications, for concentrated live loading, vary widely and the assumptions as to the distribution of weight often differ considerably from the conditions which will actually be met. In view of these facts, the writer has determined to use the 24-ton engine made by the J. I. Case Company, with the dimensions and distribution of weight, approximately as given in their catalog. As the concentrated live loading will be used only in the investigation of the floor slab, we will be interested only in the weight carried on the rear wheels. In the above engine the rear wheels are three feet wide

and are spaced seven feet center to center. It is assumed that
the load of 16,000 pounds carried on each of these wheels is dis-
tributed over an area five feet laterally by three and one half
feet longitudinally. This gives for each wheel a uniform load
equal to 16,000 ÷ (5 x 3 1/2) = 915 pounds per square foot. Thus
we have for the slab two uniform loads of 915 pounds per square
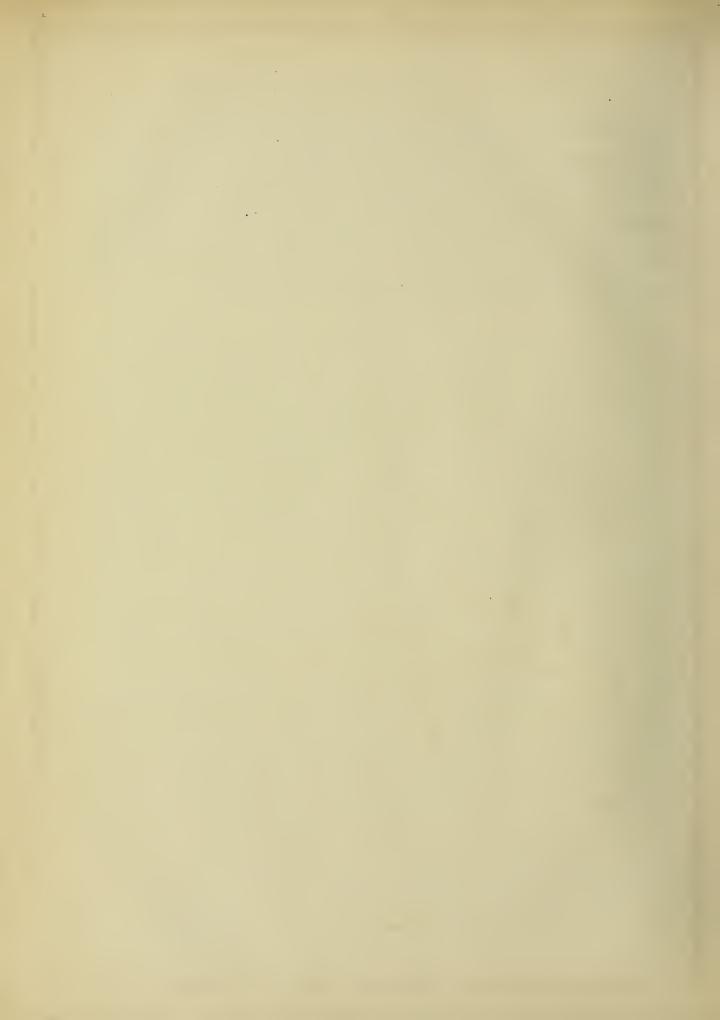foot, each five feet wide and spaced on seven feet centers.

The pressure on the foundations will be investigated
under the total dead and live load. In the excavation for the
footings, a hard layer of clay was found at about the elevation
of the stream bed which would apparently give a firm foundation;
but upon closer examination it was found that this layer was only
about one foot thick and was underlaid by a layer of loose sand
thru which a bar could easily be pushed by hand. On this account
the footings were extended to a bed of firm gravel about six
feet below the bed of the stream.

The discovery of this hard layer of clay so near the
surface, led to a further investigation of the abutments. The
stone masonry abutments, which had been used for the steel span
that was replaced by the present structure, had been examined
and found to be in such good condition that it was decided to
use them for the new bridge. The old bridge had been set upon
steel legs, and for some unknown reason very little of the load
had been placed upon the abutments. Inasmuch as they were un-
tried, the Engineer thought it possible that they were set in
the thin layer of hard clay, and proceeded to investigate. The

north abutment was found to be on a firm foundation of hard clay,
but the south one was found to be resting on the loose sand found-
ation. It was then necessary to put footings and columns under
this end of the bridge in order to take the greater part of the
load off of the old abutment. These columns can be seen in the
picture of the bridge,on the face of the abutment.
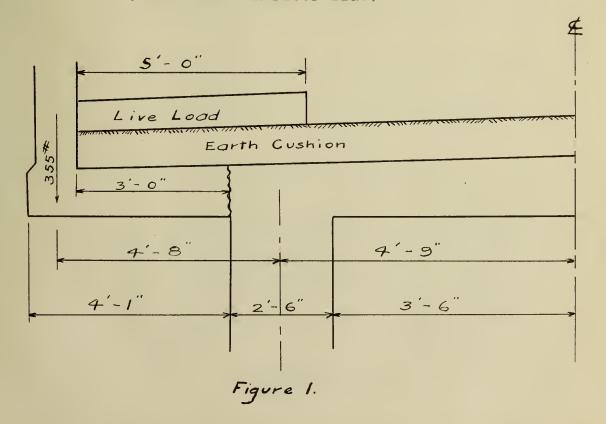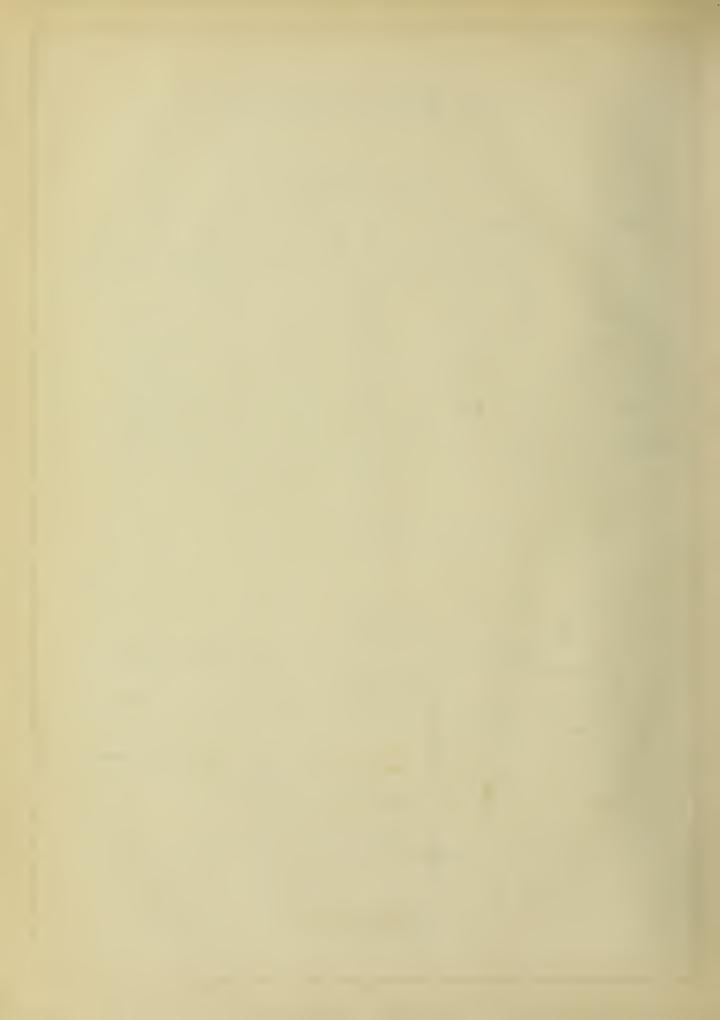
III.   COMPUTATION OF STRESSES.

A.   Table of Weights.

Part	Wt. Per linear ft.-lb.	Wt. Per sq. ft.-lb.	Total Wt. for one panel - lb.
Railing	355		18,450
Floor Slab	1955	97	101,500
Earth Cushion	900	50	46,800
Girder	1840		95,600
Largest Pier			282,750

B.   Stresses in Floor Slab.

Figure 1.

1. Section over outer edge of girder. (See Fig. I)

$$D.L.M. = -41 \times 355 - 97 \times 4.08 \times \frac{49}{2} - 50 \times 3 \times 18$$

$$= -26,950 \text{ lb. in.}$$

$$L.L.M. = -915 \times 3 \times 18 \quad -49,400 \text{ lb. in.}$$

Total $M = -76,350$ lb. in.

$$d = 5 \ 1/2 \text{ in.}$$

$$p = \frac{.60}{5.5 \times 12} = .0091$$

$$k = \sqrt{2pn \quad (pn)^2} \quad - pn$$

$$= \sqrt{2 \times .0091 \times 15 \quad (.0091 \times 15)} \quad - .0091 \times 30$$

$$= 0.403$$
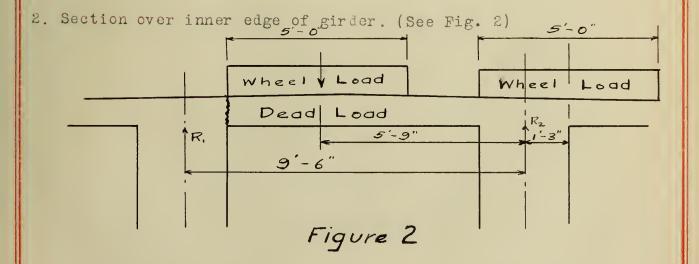
$$j = 1 - 1/3k = 1 - 1/3 \times 0.403 = .866$$
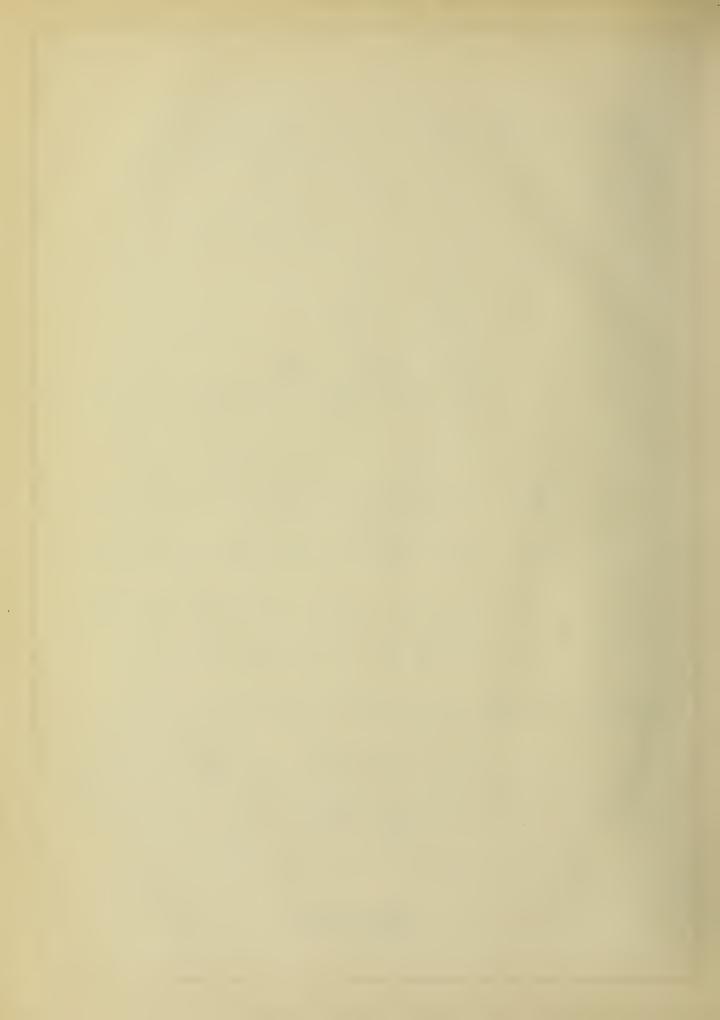
$$F_s = \frac{76,350}{.0091 \times .81 \times 12 \times (5.5)} = 26,500 \text{ lb. in.}$$

$$F_c = \frac{76,350}{1/2 \times .403 \times 91 \times 12 \times (5.5)} = 1,150 \text{ lb. in.}$$

$$V = 355 + 97 \times 4.08 + 50 \times 3 + 915 \times 3 = 3650$$

$$v = \frac{3650}{5 \ 1/2 \times 12} = 55.4 \text{ lb. per sq. in.}$$

2. Section over inner edge of girder. (See Fig. 2)

Figure 2

The moment is negligible.

Shear - Place engine load as shown in Fig. 2

Live Load - M_{R_2} = 5.75 x 915 x 5 - 1.25 x 915 x 5

$-9.5R_1$

L.L.R_1 = 2060

Total R_1 = V = 3840

$$v = \frac{3840 \quad (255 \quad 6.58 \times 97 \quad 5.5 \times 50)}{6.3 \times 12}$$

= 34 lb. per sq. in.

3. Section at center of slab

D.L.R_1 = 1780

D.L.M. = 1780 x 57 - 355 x 113 - 97 x 10.08 x 60.5

$-50 \times 9 \times \frac{108}{2}$

= -22,150 lb. in.

Stresses due to negative moment - Dead Load only
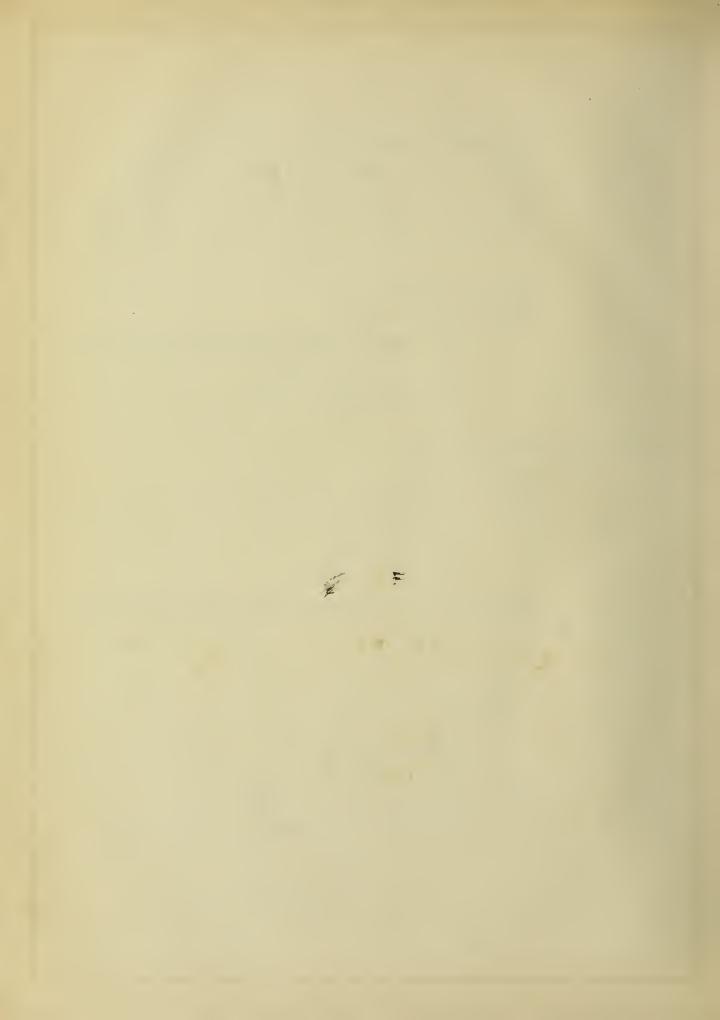
d = 7 1/2"

d'= 1 1/2"

$\frac{d'}{d}$ = 0.2

$p = \frac{.30}{12 \times 7\ 1/2} = 0.00666$

$K^2 + 2n\ (p + p')k = 2n\ (p + p'\frac{d'}{d})$

$K^2 + 30\ (.01)K = 30\ (.0033 + .00666 \times 0.2)$

K = 0.254

$\frac{C'}{C} = \frac{2p'n\ (K - \frac{d'}{d})}{k^2}$

$$= \frac{30 \times .00666 \,(.254 - .2)}{.0641} = 0.169$$

$$z = \frac{1/3k + \frac{d'C'}{d\,C}}{1 + \frac{C}{C}} \times d$$

$$= \frac{1/3 \times 0.254 + 0.2 \times 0.169}{1 \quad 0.169} \times 7.5 = 0.76$$

$$j = 1 - \frac{z}{d} = 1 - 0.102 = 0.898$$
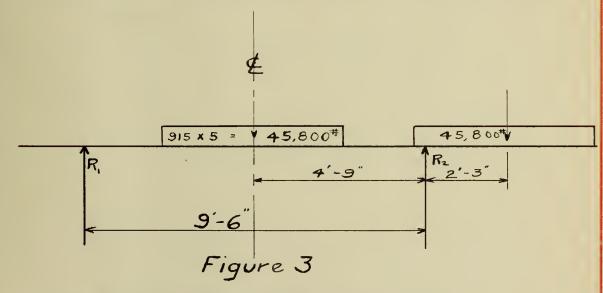
$$F_S = \frac{22,150}{.0033 \times .898 \times 12 \times (7.5)^2}$$

$$= 11,100 \text{ lb. per sq. in.}$$

$$F_c = \frac{22,150}{1/2 \times .254 \times .898 \times 12 \times (7.5)^2}$$

$$= 290 \text{ lb. per sq. in.}$$

L.L.M._ Place left wheel at center as shown in Fig. 3

Figure 3

$$\Sigma M_{h2} = 4.75 \times 4580 - 2.25 \times 4580 - 9.5 R_1$$

$$R_1 = 1200 \text{ lb.}$$

$$M = 1200 \times 57 - 915 \times 2.5 \times 15$$

$$= 34,800 \text{ lb. in.}$$

Total M = 12,050 lb. in.

As this moment is so low, no great error will be introduced by neglecting the compression steel and assuming that $j = 0.87$ and $k = 0.37$.
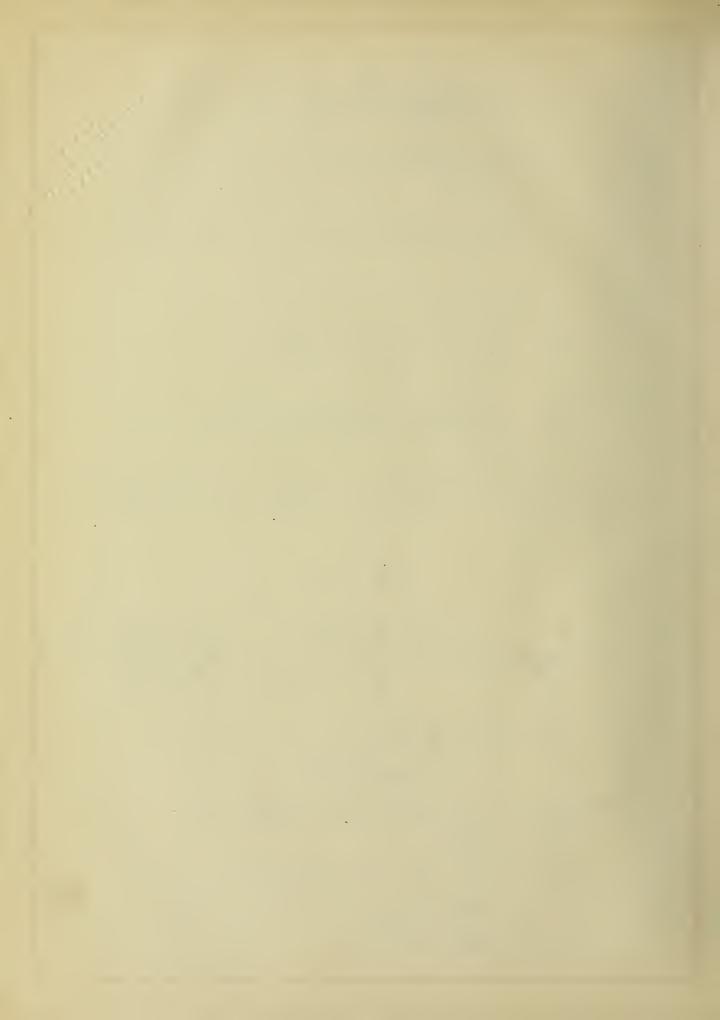
$$F_s = \frac{12,050}{.0066 \times .87 \times (7.5)^2 \times 12}$$

$$= 3100 \text{ lb.}$$

$$F_c = \frac{12,050}{1/2 \times .37 \times 87 \times (7.5)^2 \times 12}$$

$$= 135 \text{ lb.}$$

The shear at this point is negligible.

C. Stresses in the Girder.

1. Section at center of girder. (See Fig. 4)

D.L.Moment.

$$w = 1840 + 355 + \frac{1955}{2} + \frac{900}{2}$$

$$= 3620$$

$$M = 1/8 \text{ Wl}$$

$$= 1/8 \times 3620 \times (48.66)^2 \times 12$$

$$= 12,880,000 \text{ lb. in.}$$

L.L.Moment
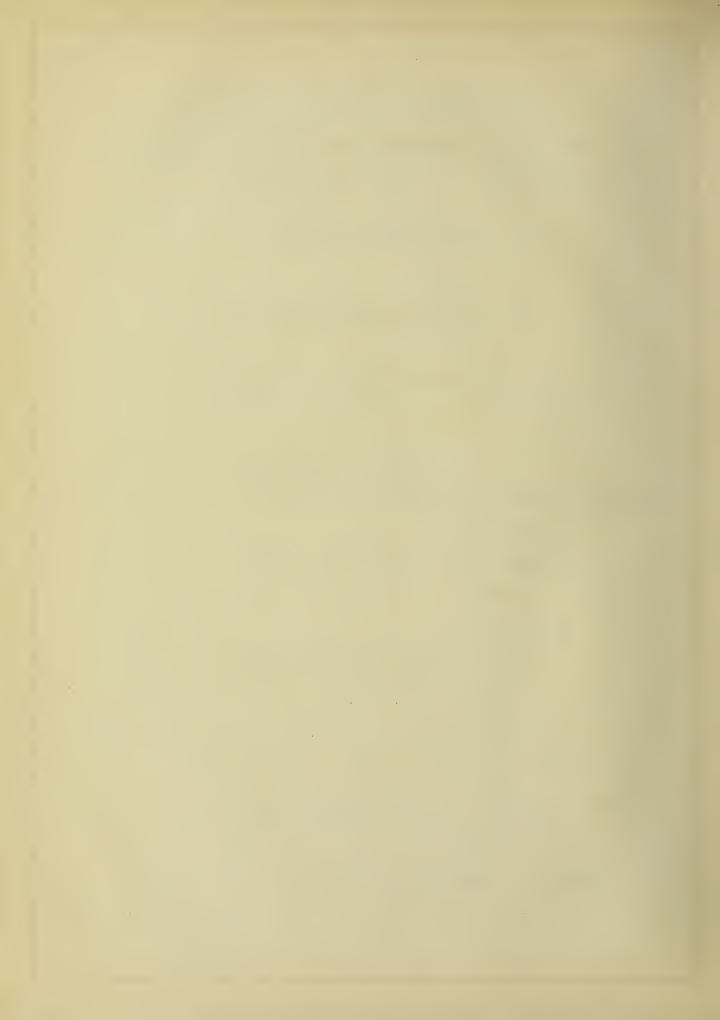
$$w = 125 \times \frac{18}{2}$$

$$= 1125$$

$$M = 1/8 \times 1125 \times (48.66)^2 \times 12$$

$$= 4,000,000 \text{ lb. in.}$$

Total $M = 16,880,000$ lb. in.

Figure 4

$$d' = \frac{2 \times 7}{2} = 4\ 1/2 \text{ in.}$$

$$p = \frac{15 \times (1\ 1/4)^2}{47 \times 30} = .0166$$

$$p' = \frac{10 \times (1\ 1/4)^2}{47 \times 30} = .0111$$

$$\frac{d'}{d} = \frac{4.5}{47} = .096$$

$$k^2 + 2n\ (p + p')k = 2n\ (p + p'\frac{d'}{d})$$

$$K^2 + 30\ (.0166 + .0111)k = 30(.0166 + .0111 \times .096)$$

$$k^2 + .831k = .531$$

$$k^2 + .831k = (.415)^2 = .531 + (.415)^2$$

$$k + .415 = \sqrt{.703} + 0.839$$

$$k = 0.424$$

$$\frac{C'}{C} = \frac{2\ p'n\ (k - \frac{d}{d})}{k^2}$$

$$= \frac{2 \times .0166 \times 15\ (.424 - .096}{.179}$$

$$= 0.914$$

$$z = \frac{\frac{1}{3}k + \frac{d'C'}{d}C}{1 + \frac{C'}{C}} \times d$$
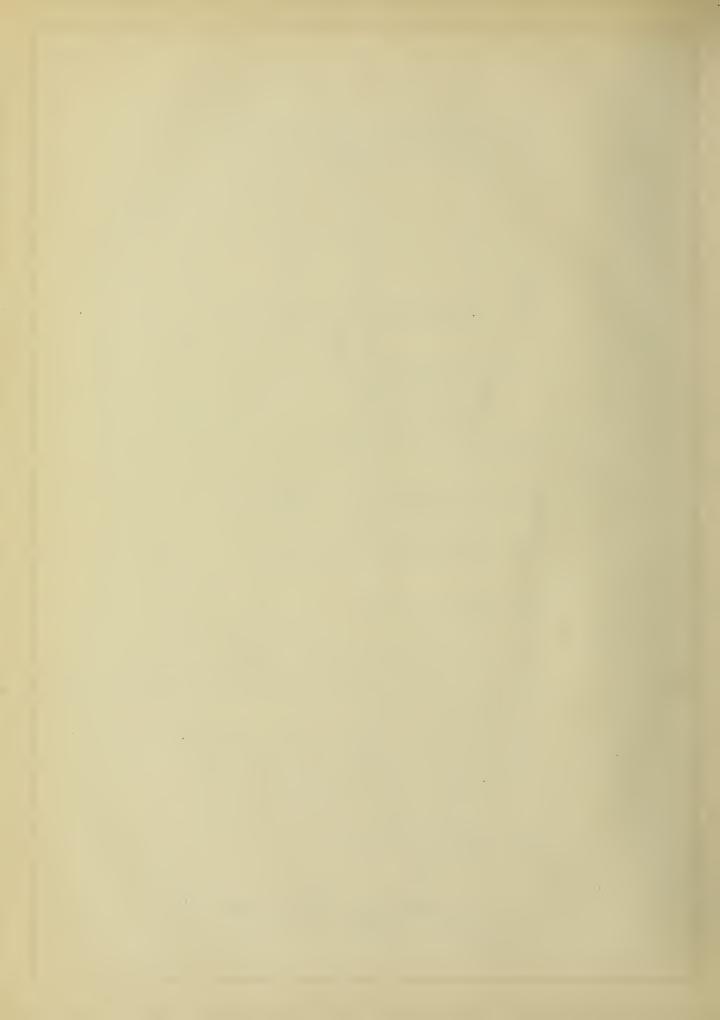
$$= \frac{\frac{1}{3} \times .424 + .096 \times .914}{1 + .914} \times 47$$

$$= 5.72 \text{ in.}$$

$$j = (1 - \frac{z}{d}) = (1 - \frac{5.72}{47}) = 0.88$$

$$F_S = \frac{M}{pjbd^2}$$

$$= \frac{16,880,000}{.0166 \times .88 \times 30 \times (47)^2}$$

$$= 17,400 \text{ lb. per sq. in.}$$

$$F_c = \frac{F_S k}{n(1-k)}$$

$$= \frac{17,400 \times .424}{15(1-.424)}$$

$$= 854 \text{ lb. per sq. in.}$$

$$F_S = n \frac{k - \frac{d'}{d}}{k} \times F_c$$

$$= 15 \frac{.424 - .096}{.424} \times 854$$

$$= 9,900 \text{ lb. per sq. in.}$$

2. Section over edge of Pier - shearing stresses.

$$R_1 = (3620 + 1125) \times \frac{52}{2}$$

$$= 123,000 \text{ lb.}$$

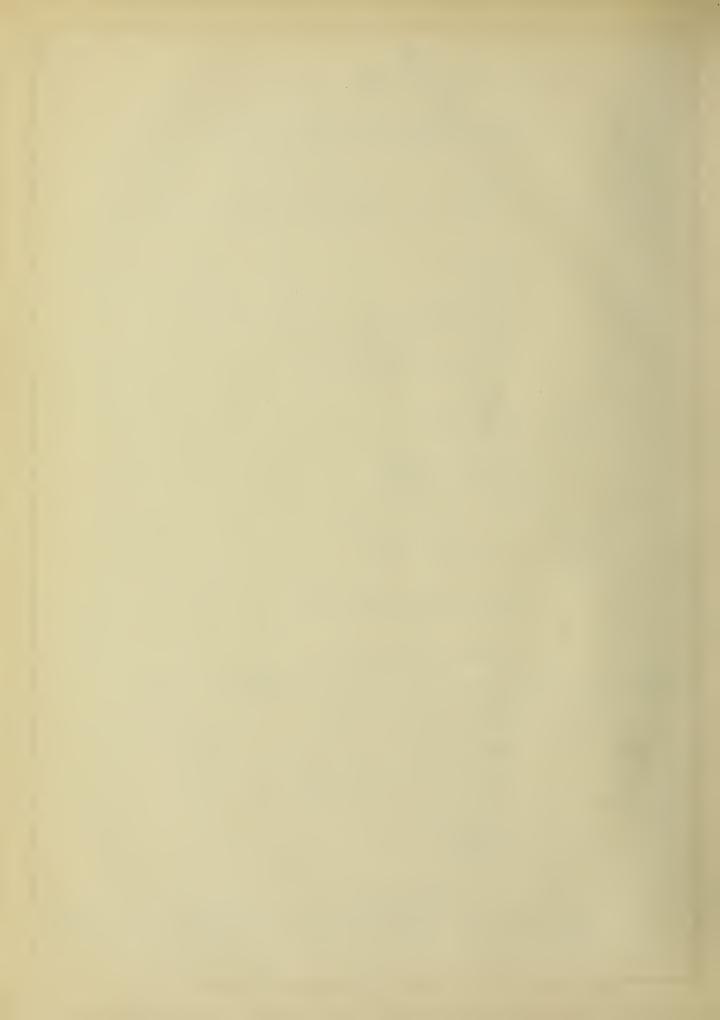$$V = (3620 + 1125) \times \frac{48.66}{2}$$

$$= 115,200 \text{ lb.}$$

$$v = \frac{115,200}{47 \times 30}$$

$$= 82 \text{ lb. per sq. in.}$$

Assume that the concrete takes 40 lb. per sq. in. shearing
stress.

82 - 40 = 42 lb. per sq. in.= shear to be taken by

stirrups.

The stirrups at the end are 1/2 inch square bars on 9 in.
centers.

Area per linear foot of girder = $(1/2)^2$ x 2 x $\frac{12}{9}$

= 0.66 sq. in.

$F_s = \frac{42 \text{ x } 30 \text{ x } 12}{.66}$

= 22,900 lb. per sq. in.

Assume that the stirrups take a stress of 18,000 lb. per
sq. in.

v_s = Shear taken by stirrups.

= $\frac{18,000 \text{ x } .66}{30 \text{ x } 12}$ = 33 lb. per sq. in.

v_c = Shear in concrete

= 82 - 33 = 49 lb. per sq. in.

3. Section 6 feet from center of Pier - shearing stresses.

V = 123,000 - 6 x (3620 + 1125) = 94,500

v = $\frac{94,500}{30 \text{ x } 47}$ = 67 lb. per sq. in.
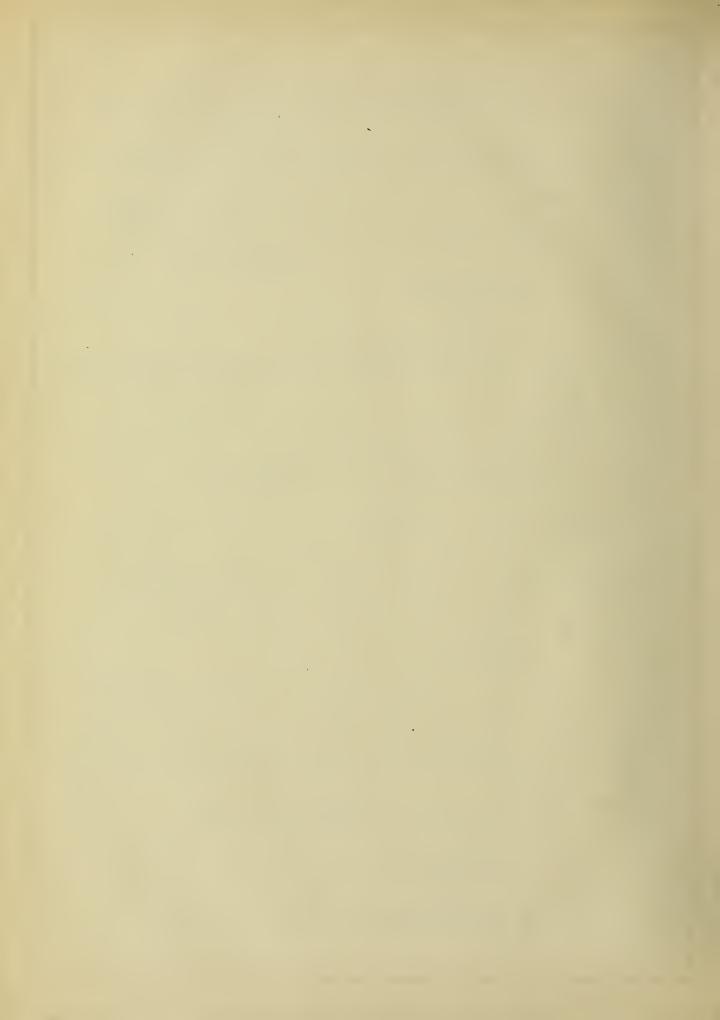
Assume that concrete takes 40 lb. per sq. in. shear

v_s = 67 - 40 = 27 lb. per sq. in.

The stirrups here are 1/2 inch square bars on 15 in.
centers.

Area = $(1/2)^2$ x 2 x $\frac{12}{15}$ = .40 sq.in.

$F_s = \frac{27 \text{ x } 30 \text{ x } 12}{.40}$

= 24,300 lb. per sq. in.

Assume stirrups take 18,000 lb. per sq. in.

$$v_s = \frac{18,000 \times .40}{30 \times 12} = 20 \text{ lb. per sq. in.}$$

$$v_c = 67 - 20 = 47 \text{ lb. per sq. in.}$$

4. Section 10 feet from center of Piers - shearing stress.

$$V = 123,000 - 10 \times (3620 \quad 1125) = 75,500 \text{ lb.}$$

$$v = \frac{75,500}{30 \times 47} = 53.5 \text{ lb. per sq. in.}$$

Assume that the concrete takes 40 lb. per sq.in. shear

$$v_s = 53.5 - 40 = 13.5 \text{ lb. per sq. in.}$$

The stirrups here are 1/2 inch square bars on 27 in. centers.

$$\text{Area} = (1/2)^2 \times 2 \times \frac{12}{27} = 0.222$$

$$F_s = \frac{13.5 \times 12 \times 30}{.222}$$

$$= 21,900 \text{ lb. per sq. in.}$$

Assume that the stirrups take 18,000 lb. per sq. in.

$$v_s = \frac{18,000 \times .222}{12 \times 30} = 11.1 \text{ lb. per sq. in.}$$

$$v_c = 53.5 - 11.1 = 42.4 \text{ lb. per sq. in.}$$
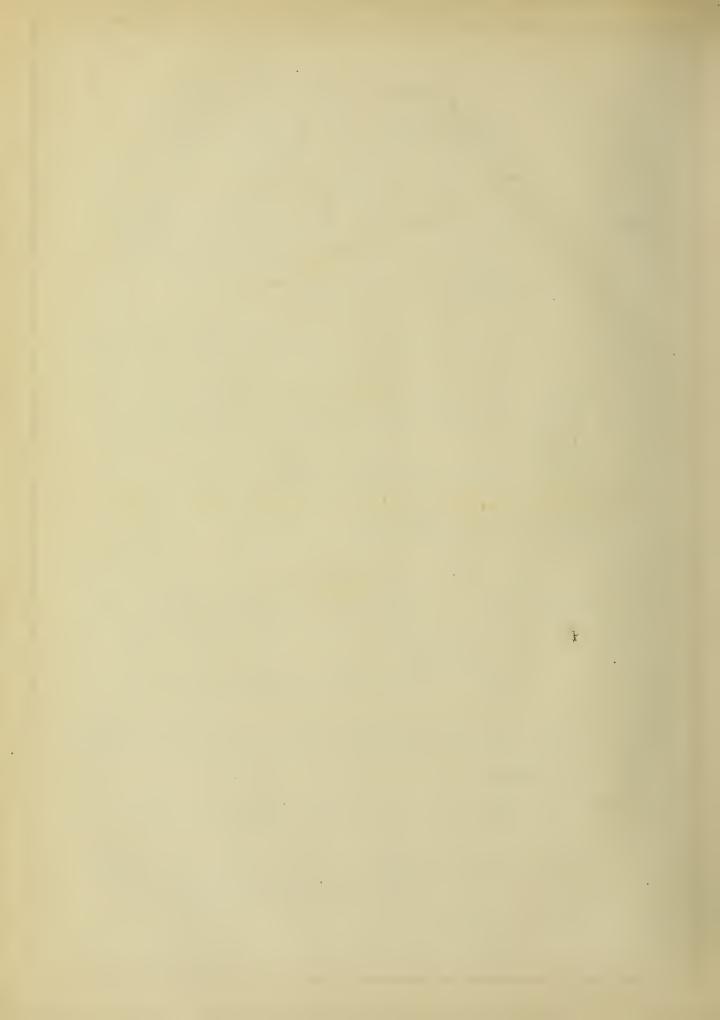
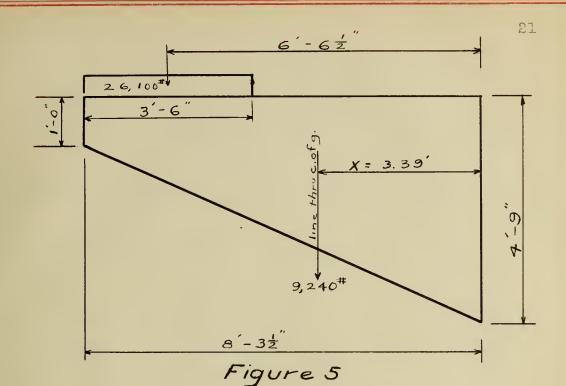D. Stresses in Cantilever at end of Bridge. (See Fig. 5)

D.L. Moment

$$W = \frac{1 + 4.75}{2} \times 2.5 \times 8.29 \times 155 = 9,240 \text{ lb.}$$
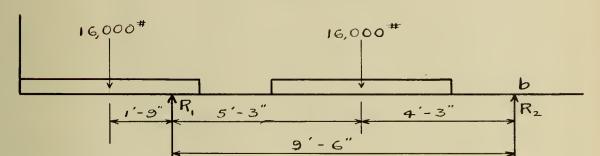
$$X = \frac{1 \times 8.29 \times \frac{8.29}{2} + 1/2 \times 3.75 \times 8.29 \times \frac{8.29}{3}}{1 \times 8.29 + 1/2 \times 3.75 \times 8.29}$$

$$= 3.39 \text{ ft.}$$

$$M = WX = 9,240 \times 3.39 \times 12 = 375,000 \text{ lb. in.}$$

Figure 5

Figure 6

I.L.Moment

$$\Sigma M_b = 16,000 \text{ x } (4.25 - 11.25) - 9.5R_1 = 0$$

$$R_1 = 26,100$$

$$M = 26,100 \text{ x } 6.54 \text{ x } 12 = 2,050,000 \text{ lb. in.}$$

Total $M = 2,425,000$ lb. in.

$$A = 4 \text{ x } (1/2)^2 = 1 \text{ sq. in.}$$

$$d = 55 \text{ in.}$$

$$p = \frac{1}{30 \text{ x } 55} = .0.0061$$

$$k = \sqrt{2pn \; (pn)^2} - pn$$

$$= \sqrt{30 \text{ x } .00061 \quad (.00061 \text{ x } 30)^2} - (.00061 \text{ x } 30)$$

$$= .12$$

$$j = 1 - 1/3k = 1 - 1/3 \times .12 = 0.96$$

$$F_s = \frac{2,425,000}{.00061 \times .96 \times 30 \times (55)^2} = 45,700 \text{ lb. per sq. in.}$$

$$F_c = \frac{M}{1/2 \; kjbd^2} \qquad \frac{2,425,000}{1/2 \times .12 \times .96 \times 30 \times (55)^2}$$

$$= 464 \text{ lb. per sq. in.}$$

Shear

The maximum shear in the cantilever occurs when the rear wheels of the engine are directly above the edge of the abutment.

$V = 25,500$ lb. as shown above

$$v = \frac{25,500}{30 \times 55} = 15.5 \text{ lb. per sq. in.}$$

E. Bearing on Foundations.
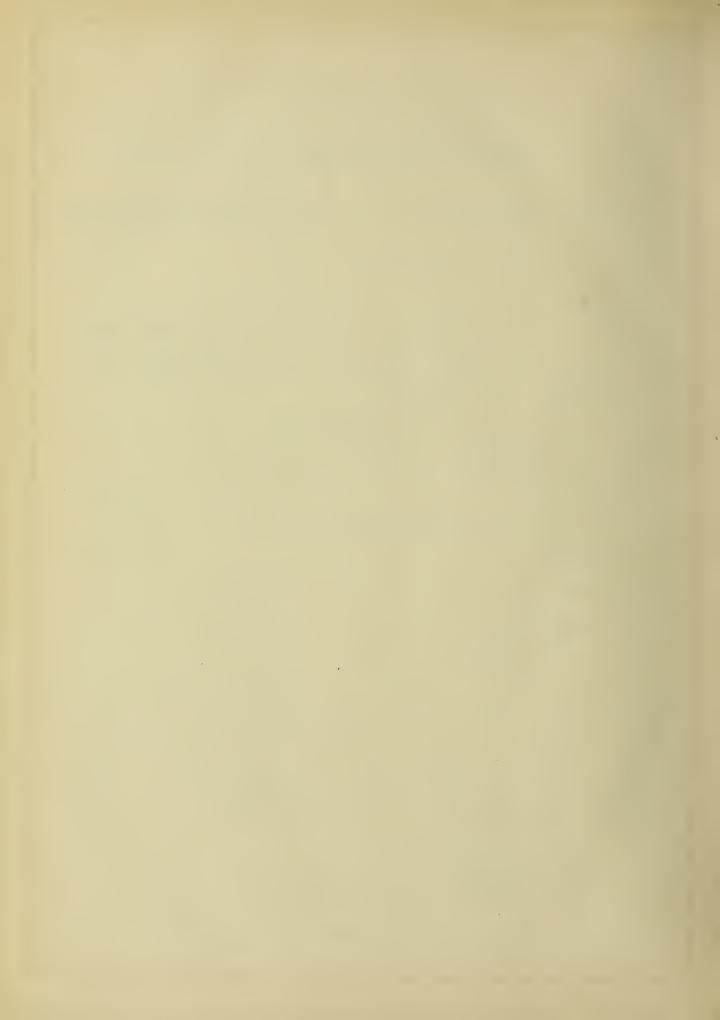
Dead Load

Railings -	- 36,900 lb.
Floor slab-	-101,500 "
Earth cushion -	- 46,800 "
Girders -	-191,200 "
Pier -	-282,750 "
Total	-659,150 "
Live Load - 125 x 18 x 52 -	-117,000 "
Total Load -	-776,150 "

Pressure on Foundations

$$P = \frac{776,550}{200} = 3,880 \text{ lb. per sq. ft.}$$

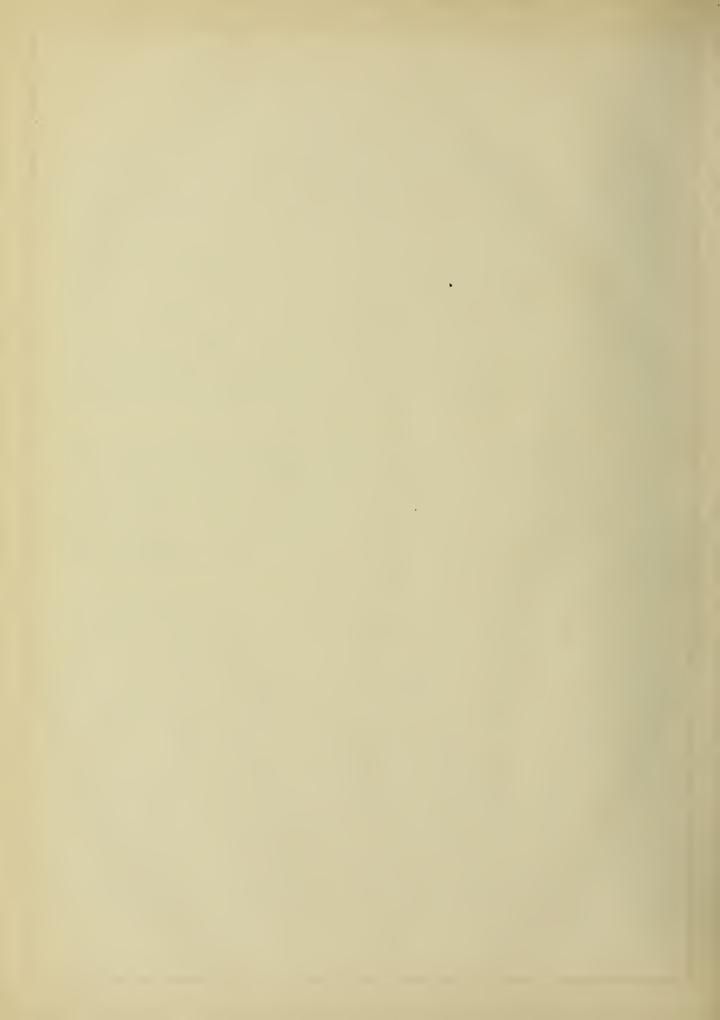NOTE: 200 sq. ft. = Area of Base of Footing.

IV. DISCUSSION.

It is apparent that the assumptions as to live loading used in this investigation differ considerably from those used in the design of the bridge. The live loading used here is admittedly severe, probably heavier than that used in standard practice. The Illinois Highway Commission's "Specifications for the Design of Steel Bridges", in the edition of April, 1913, calls for a 20 ton engine on two axles spaced ten feet apart. The writer does not know how the weights on each wheel are assumed to be distributed.

The 24 ton traction engines are not as yet very common on the roads of this State, but several large firms build them with dimensions approximating those given above for the J.I. Case engine, and it is undoubtedly true that they will soon come into more common use for hauling materials.

It is difficult to determine how the load on the large wheels will be distributed by the six inch earth cushion. The wheels are eight feet in diameter and three feet wide. The area of earth which carries the load is three feet by about one and one half feet, and a fair assumption seems to be that the load will be spread out one foot on each side of this area.

The highest stresses found in the slab, occur at a point over the outer edge of the girder when the engine wheel is placed on the side of the bridge within a foot of the railing, so that the live load on the slab is distributed to the edge of the railing.
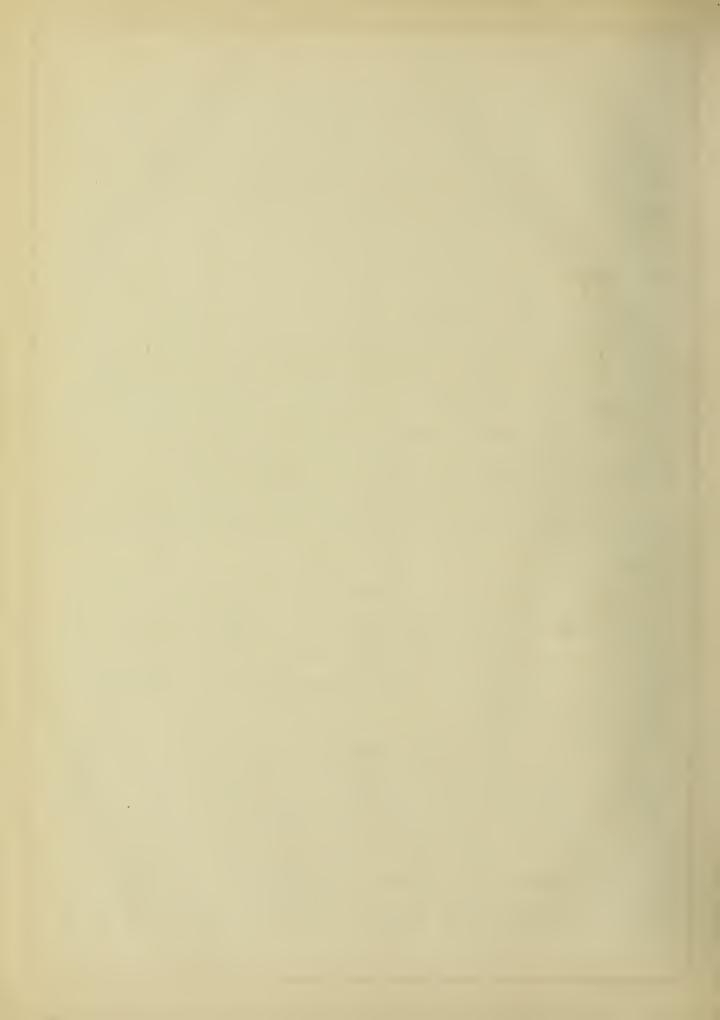
The steel stress is 26,500 pounds per square inch, the compression in the concrete is 1,150 pounds per square inch, and the shear in the concrete is 55.4 pounds per square inch. These stresses are considerably in excess of those used in good design. A steel stress of 16,000 to 18,000 pounds per square inch has become standard. Turneaure and Maurer recommends for conservative design, 650 pounds per square inch compression in concrete, but the Illinois Highway Commission uses stresses of from 800 to 1000 pounds per square inch. The unit allowable shear of 40 pounds per square inch recommended by Turneaure and Maurer is sometimes raised to 50 pounds per square inch, but the latter value is higher than that commonly used without web reinforcement.
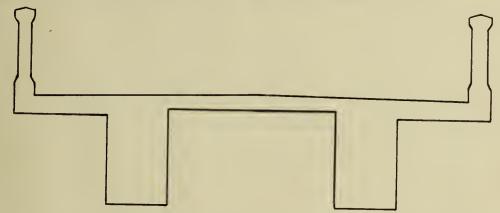
At the inner edge of the girder, the shear is the governing factor. The value of 34 pounds per square inch found here is safe.

At the center of the slab, the stresses are higher due to the negative moment caused by dead load alone, than they are under the worst conditions of live loading. In neither case are they much more than half of the allowable stresses recommended by Turneaure and Maurer.

In view of the non-uniformity of the stresses in the slab, two suggestions are offered for the improvement of the design. The first is to increase the distance between girders, which would reduce the moment and shear at the support and increase the moment at the center. The disadvantage of such a change is that the width of piers would need to be increased,

thus adding to the cost of the bridge. The second suggestion is
to increase the depth of slab and area of steel at the support,
thus reducing the unit stresses at this point, and to decrease the
depth of slab and steel area at the center, thus increasing the
unit stresses at this point. A section similar to that shown be-
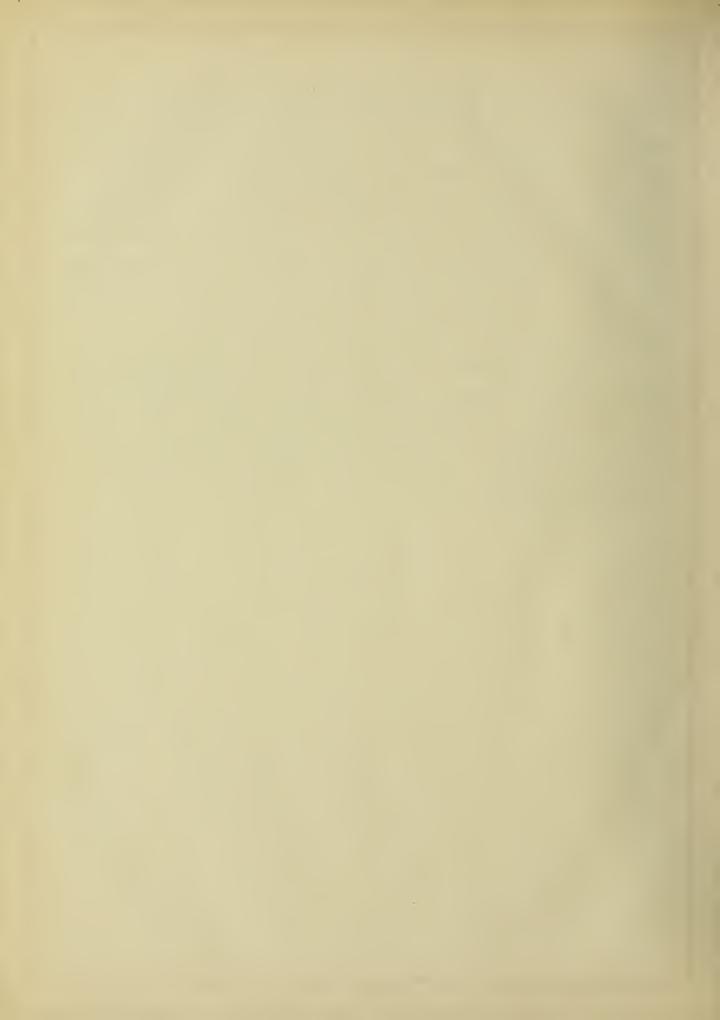low carries out the idea of the second suggestion.

　　　　　The direct tensile and compressive stresses found in
the main girder, altho slightly higher than those used in con-
servative design, are not excessive. There is some uncertainty
as to the way in which the web stresses are divided between the
steel and concrete. The two assumptions made above show the
range of stresses which theoretically can be expected. The com-
putations show that the shear in the concrete is between 40 and
50 pounds per square inch, and that the stress in the stirrups
is between 18,000 and 24,300 pounds per square inch. Regardless
of how these stresses are divided, they are in either case high,
showing that fewer stirrups were used than the number required
for good design.

　　　　　The cantilevers at the ends of the bridge were evidently
not designed to carry the full load. They were cast without

bottom forming,so as to rest directly upon the ground which will carry some of the load. In view of the fact that the excavation here is not below the frost line , there is little certainty that the ground will not settle, leaving the entire load to be carried by the cantilever action, which would be very apt to cause failure. By doubling the amount of steel in this member , thus adding about 100 pounds to the total weight of steel in the bridge, the design could have been made safe against this possibility.

The footings of the piers are on a foundation of firm gravel, which according to "Baker's Masonry Construction" has a safe bearing power of from 8 to 10 tons per square foot. With a pressure of about 2 tons per square foot as found above, the safety factor is four.

V. CONCLUSIONS.

In drawing conclusions from the above investigation, the fact must be kept in mind that the loadings used are probably more severe than those for which the bridge was designed.

In no case have the stresses exceeded the elastic limit of the steel or the concrete; in some cases they are dangerous, not allowing the safety factor which is usually used, and in general they are in excess of the allowable stresses used in good design.